Comparison of multi-storey cross-laminated timber and timber frame buildings by in situ modal analysis

Citation for published version:
Reynolds, T, Casagrande, D & Tomasi, R 2016, 'Comparison of multi-storey cross-laminated timber and timber frame buildings by in situ modal analysis', *Construction and Building Materials*, vol. 102, no. 2, pp. 1009-1017. https://doi.org/10.1016/j.conbuildmat.2015.09.056

Digital Object Identifier (DOI):
10.1016/j.conbuildmat.2015.09.056

Published In:
Construction and Building Materials

Comparison of multi-storey cross-laminated timber and timber frame buildings by in-situ modal analysis

This is the author's accepted version - Full paper at http://dx.doi.org/10.1016/j.conbuildmat.2015.09.056

Thomas Reynolds[a,], Daniele Casagrande[b], Roberto Tomasi[b]

[a] *University of Cambridge, Department of Architecture, 1 Scroope Terrace, Cambridge CB2 1PX*
[b] *Department of Civil, Environmental and Mechanical Engineering, University of Trento*

Abstract

The dynamic properties of two common structural systems for multi-storey timber buildings are compared through in-situ testing of completed buildings. The two five-storey buildings examined are identical except for their structural system, which in one is sheathed stud-and-rail timber construction, and in the other a cross-laminated timber panel system. Both also have a reinforced-concrete core located at the centre of one edge of the rectangular plan of each building. An output-only modal analysis method was used to identify the modal properties of the buildings: the random decrement technique was applied to the stochastic measured response, and then the time-domain random decrement signature was used for modal analysis by the Ibrahim Time Domain method. The natural frequencies, damping ratios and mode shapes of the first three vibration modes of the buildings were identified, and compared with those modeled based on the properties of the core and timber walls. The variations in properties between the two buildings are discussed. The two structures show very similar natural frequencies and damping ratios, suggesting that they could perhaps be considered as the same class of building in design for lateral movement.

Keywords: timber, multi-storey, modal anlaysis, damping

1. Introduction

Timber construction systems have in recent years been increasingly used in multi-storey residential construction in Europe, which represents a change from the one- and two-storey houses responsible for the bulk of structural use of timber over the last century. An example of this is the choice of the Trentino social housing company ITEA to build two five-storey multi-family apartment buildings in Trento (Italy) using two different timber wall systems: timber frame (TF) and cross laminated timber (CLT). An external view of the two buildings is shown in Figure 1. The design and the construction of these two buildings are the result of an international collaboration between ITEA and the Canadian social housing company Quebec Societè D'Habitation in order to evaluate and compare timber construction systems for social housing from the two countries. Since the two buildings have identical floor plans, layouts and finishes and differ only in their structural system, they present a unique opportunity for comparison of the two key forms of timber construction in economic and engineering terms. One parameter for comparison is their dynamic behaviour under lateral loads.

For the construction of larger and taller timber buildings, it is necessary to understand the dynamic behaviour of timber structural systems. This is because movements, which may be acceptable for small heights and spans, are magnified in large structures, and may cause discomfort to building occupants, damage to non-structural elements, or increased loads on elements. These structures behave nonlinearly, due to the embedment behaviour of the nails and screws in their connections, as well as the rigid-body rotations of panels, and so their dynamic behaviour depends on the amplitude of the exciting force. Research must therefore address dynamic behaviour in both the serviceability limit state, under loads such as wind and footfall, and for the ultimate limit state, under seismic or wind loads. In order to model and predict the response of structures to such dy-

Email address: `tpsr2@cam.ac.uk` (Thomas Reynolds)

Figure 1: Photograph of the buildings looking from the south east

namic loads, a knowledge of natural frequency, damping and mode shape is required. This study records those values under the reversible movements associated with the serviceability limit state.

The objective of this study is to obtain the modal parameters under small vibrations for these two buildings, both to fill a gap in knowledge of the modal properties of timber-concrete hybrid multi-storey buildings, and to compare the structural systems of the two buildings. For these reasons the authors carried out an in-situ experimental modal analysis, to measure natural frequencies, damping ratios and mode shapes of each building.

When timber is to be used in larger buildings, it is likely to be used in conjunction with other materials, such as reinforced concrete or steel, and so the interaction between timber and concrete structural systems, observed in these buildings, is of interest. A further objective is therefore to investigate the dynamic interaction between the concrete core and the timber walls and to study how these two different structural systems influence the global response of the entire building. In this study, natural frequencies were calculated based on the structural action of the concrete core alone, since that was considered to be readily predictable using established methods. The difference between the natural frequencies predicted using the core alone and those measured for the entire building could then be used to give an insight into the contribution of the timber structure in each building in each mode.

2. Background

These buildings represent two widely-used, but fundamentally different forms of timber construction. The sheathed stud-and-rail system of the north building, commonly referred to as light timber frame (TF) has been used for many years and can be used efficiently with prefabrication of elements, since fully sealed and insulated panels can be constructed which can be lifted into place and connected to each other with hand tools on site. In sheathed timber stud walls, the lateral stability is provided by the sheathing, which is either nailed or screwed to the studwork. The lateral stiffness is therefore dependent on the stiffness of the connection provided by the nails or screws. This form of construction is extremely lightweight, which gives it the potential to be used in expansion and retrofit of buildings, as well as in new construction.

Cross-laminated timber (CLT), used to form the timber shear walls in the south building, is a comparatively modern building component. Again, it can be highly prefabricated, and used with computer controlled cutting techniques to pre-cut openings for windows, doors and services to a high degree of accuracy. In CLT structural systems, connections again play an important part in the resistance of the structure. Under large-amplitude load, such as seismic loading, the panels are generally relatively stiff in comparison with the connections between them. Under smaller-amplitude in-service wind loads, however, it may be that the gravity loads and friction between panels are not overcome, so that very little relative movement occurs between panels, and the stiffness of the panels themselves assumes greater importance.

The dynamic response of a multi-storey building using a sheathed stud-and-rail timber frame construction system was studied on the six-storey Timber Frame 2000 test building by Ellis and Bougard (2001). They tested the building under both ambient vibration and using a rotational shaker. Ambient vibration tests have also been carried out on a seven-storey CLT building (Reynolds et al., 2015). Aside from these studies, however, the majority of research into the dynamic response of such structures has been under the irreversible deformations associated with seismic loading. The behaviour of this form of construction has been studied through experiment and numerical modeling (van de Lindt et al., 2011; Folz and Filiatrault, 2001; Filiatrault et al., 2009; Judd and Fonseca, 2005; Tomasi et al., 2014), and through full nonlinear finite element modeling (He et al., 2001), and used for the creation of simplified design rules for calculating the appropriate stiffness of the shear wall systems (Casagrande et al., 2015).

The stiffness of CLT systems has been studied both in terms of the overall elastic properties of the panels themselves (Gsell et al., 2007), and those of complete structural systems including connections (Vessby et al., 2009). The seismic performance of the components and systems in CLT buildings was widely studied (Gavric et al., 2015; Fragiacomo et al., 2011), and full-scale shaking table tests of buildings were carried out under the SOFIE project (Ceccotti et al., 2013). Under seismic loads, the movement is dominated by deformation in the connections, which are relatively flexible in comparison with the panels themselves. The response of the buildings can be expected to be different under the smaller vibration due to wind load since, in these conditions, the forces are transferred between panels primarily by friction and normal edge forces, and connections may be much more lightly loaded. Further research is therefore required to determine those parameters.

Modal analysis of structures can be carried out using records of both excitation force and output vibration, expressed as either acceleration, displacement or velocity. In these tests, however, the building owners were not willing to allow a shaker or other device to be attached to the structure to provide the necessary excitation force, and so for this test series, an output-only modal analysis technique was used. Such techniques have been applied to buildings and bridges (Brownjohn et al., 2010; Cunha and Caetano, 2006; Magalhães et al., 2012; Foti et al., 2012), and rely on the stochastic excitation produced by the ambient conditions of wind.

In this study, we focus on the small-amplitude vibration observed in these buildings under ambient wind conditions. The modal properties observed under such conditions are most directly applicable as an indication of the sensitivity of these buildings, and taller multi-storey buildings of similar form, to wind-induced vibration. They also provide an estimate of the as-built dynamic properties of the structure which, bearing in mind the expected variation in deformation mechanisms described above, may be a useful indication of the dynamic properties relevant to seismic excitation. Since tests on completed structures must be limited to reversible deformations, this type of test is an important tool for assessment of their sensitivity to earthquakes.

3. Description of test buildings

The tested buildings have 5 storeys with a total height of 15.6 m and are approximately rectangular in plan (14.7 m x 18.5 m) , with the reinforced concrete stair core at the centre of the long edge. A plan of the buildings, indicating their structure, is shown in Figure 2, and an elevation showing the floor levels is given in Figure 3.

In the north (TF) building, the walls are made with 120 mm x 180 mm solid timber studs at a spacing of 625 mm. Oriented Strain Board (OSB/3) sheathing panels, 18 mm thick, on both sides of the wall were used to provide lateral stability, connected to the wood frame by means of ring nails (3.1 mm x 60 mm) with a spacing on the panel perimeter of 50 mm. A timber-concrete composite structure was used for the floors, connecting 100 mm x 200 mm glulam beams (with a spacing of 58 cm) to a 50 mm thick concrete slab with vertical screws. The floors are connected to the core by steel bars designed to transfer seismic forces. The bars are cast into the 50 mm concrete slab on top of the floor, and into the concrete core.

In the south (CLT) building, the walls were made with 5-layer CLT panels with a thickness of 153 mm for the ground and the first floor and of 133 mm for the upper floors. Floors were made with 5-layer CLT panel with a height of 153 mm and a width of 1.70 m transversally connected by 45°inclined screws. The CLT floor panels were screwed to 10x24 cm glulam beam which in turn were bolted to the concrete core.

Both buildings may be described as having platform construction, since each wall is interrupted at each storey level. In the CLT building, the floors are loaded perpendicular to the grain at each storey. In the TF building, the timber beams of the floor are not placed on the top plate of timber frame walls, but are connected to it with screws inclined at 45°. Perpendicular to grain loading therefore only occurs in the top and bottom plate of the walls.

In both buildings, the rigid-body rotation and the sliding of each walls were prevented by hold-downs, on the corner of the walls, and angle brackets respectively, designed to bear both wind load and seismic load. Both buildings have a 5.20 m x 5.60 m reinforced concrete core (wall thickness 200 mm), which contributes to the resistance of vertical and lateral loads working together with the timber shear walls. The buildings can therefore be described as timber-concrete hybrid structures.

The total length of shear walls running in a particular direction may be considered as an indication of the stiffness of the building in that direction. The total length of shear walls in both buildings is 22.8 m and 17.1 m along the y (north-south) and x (east-west) directions respectively. In the north-south direction, the greater total length of the two concrete walls (11.20 m) and the greater total length of the timber shear wall (22.8 m) suggests that the building would be stiffer, and therefore have a higher natural frequency, than in the east-west direction. Moreover, because of the eccentric location of the concrete core, it is expected than the translation

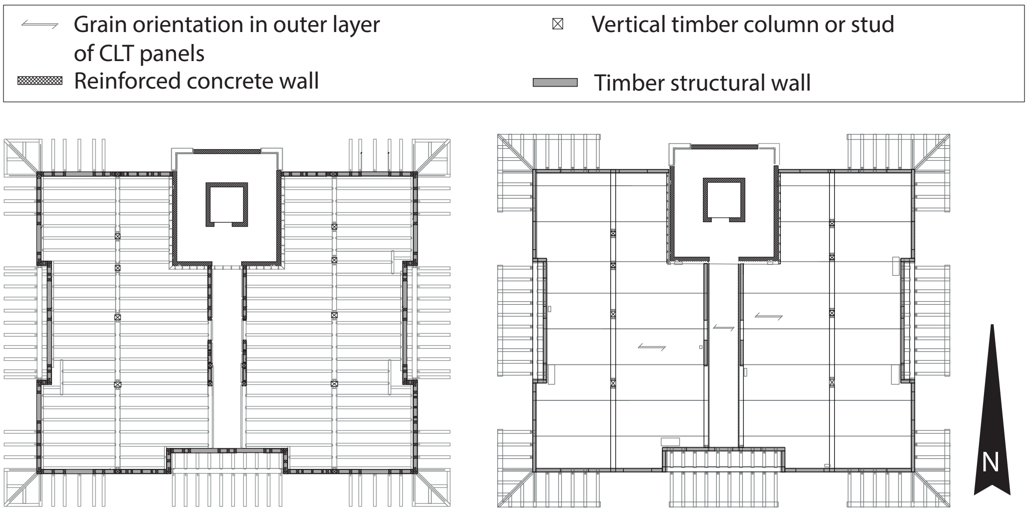

Figure 2: Plan sections of the TF (left) and CLT (right) tested buildings, showing structural elements

Figure 3: Schematic elevation of a building showing the levels for measurement of acceleration

Table 1: Weight of CLT building

	Floors (kN)	Walls (kN)	Total (kN)	Area (m^2)
Ground Floor	944.1	271.9	1216.0	223
P1	944.1	244.9	1189.0	223
P2	944.1	252.1	1196.2	223
P3	843.5	218.9	1062.4	223
Roof	366.4	84.9	451.3	352
Total weight		5114.9	kN	

Table 2: Weight of TF building

	Floors (kN)	Walls (kN)	Total (kN)	Area (m^2)
Ground Floor	1030.9	220.3	1251.2	223
P1	1030.9	204.4	1235.4	223
P2	1030.9	155.8	1186.8	223
P3	931.7	125.4	1057.1	223
Roof	345.5	71.8	417.2	352
Total weight		5147.6	kN	

mode shape in the east-west direction will be coupled with the torsional mode shape. Because the symmetry of the building in the north-south direction, no torsional effect was expected in this direction.

The weight of each building is summarised in Table 1 and Table 2. It can be seen that the mass of the two buildings is almost identical, but the distribution of the mass is slightly different. The two buildings have the same non-structural elements in the floor, so any difference in mass is due to the structure and the walls. The higher mass in the walls of the CLT building, and therefore at its perimeter, would be expected to reduce the frequency of the first torsional mode of vibration. The mass in the TF building is also biased slightly more towards the higher levels, which would be expected to reduce the lateral and torsional frequencies.

4. Methods

The tests used a set of 10 piezoelectric accelerometers, mounted to measure acceleration in 2 perpendicular horizontal directions in 5 locations. The accelerometers had a sensitivity of 10 V/g, a frequency range from 0.2 Hz to 1500 Hz and a measurement range of ± 4.9 m/s^2. Two pairs of accelerometers (A1 to A4) were kept in position to provide a pair of possible reference locations, while the other three pairs (from A5 to A10) were moved to measure at the four corners of the buildings at each of the four above-ground floors. In order to record the signals from all four corners in both direction for all the floors (the roof was excluded), 5 tests were carried out for each building. A sampling frequency of 600 Hz was adopted with an appropriate anti-aliasing filter. The mean signal duration was approximately 30 minutes.

In Table 3 the location and the direction of each channel is shown for all 5 test set-ups. In Figure 4 the accelerometer locations and directions for test set-up number 2 are shown. The measured data from each of these tests was first filtered using a 5-pole Butterworth bandpass filter, with pass band between 0.1 Hz and 30 Hz. It was then analysed using the random decrement technique. This resulted in a set of coordinated random decrement signatures for each test.

The random decrement technique (H Cole, 1968) averages a group of data segments, each starting at the point where the signal crosses a chosen trigger level. Segments of data were rejected, and not used in the averaging process, if their standard deviation was more than 10% higher or lower than the standard deviation of the complete signal. This helped to ensure stationarity of modal properties in the segments being averaged, since the segments being averaged all had similar amplitude, and would therefore be expected to exhibit similar modal properties.

The Ibrahim Time Domain (ITD) method was then used to extract modal parameters from the set of random decrement signatures, giving a set of natural frequencies, damping ratios and partial mode shapes for each test. The partial mode shapes could then be combined, by scaling them so that the amplitude of the reference channels in that mode was the same for the partial mode shapes from each test.

The sensitivity of the results to the random decrement trigger level was investigated to choose an appropriate level. Figure 5 shows the set of results for the first 10 channels measured for the CLT building. The random decrement trigger level was set as a multiple of the standard deviation for that record. It can be seen in the figure that a change in the multiplier has an effect on the natural frequency and damping ratio calculated by the ITD method. First, the modal confidence factor deviates substantially from unity for a multiplier larger than 1.6, which, along with the low number of samples for averaging, suggests these data are unreliable. Below 1.6, the MCF is consistently very close to unity, the number of averages is over 2000 until below 0.7, and the estimates of natural frequency are stable. This suggests

Table 3: The five tests carried out on each building (C: Corner, F: Floor, D: Direction, SW: South-West, NW: North-West, SE: South-East, NE: North-East, S: South, E: East, W: west, N: North

Accel.	Test Setup #1			Test Setup #2			Test Setup #3		
	C	F	D	C	F	D	C	F	D
A1	SW	3rd	S	SW	3rd	S	SW	3rd	S
A2	SW	3rd	W	SW	3rd	W	SW	3rd	W
A3	SW	4th	S	SW	4th	S	SW	4th	S
A4	SW	4th	W	SW	4th	W	SW	4th	W
A5	SE	4th	S	SE	3rd	S	SE	2nd	S
A6	SE	4th	W	SE	3rd	W	SE	2nd	W
A7	NE	4th	S	NE	3rd	S	NE	2nd	S
A8	NE	4th	W	NE	3rd	W	NE	2nd	W
A9	NW	4th	S	NW	3rd	S	NW	2nd	S
A10	NW	4th	W	NW	3rd	W	NW	2nd	W

Accel.	Test Setup #4			Test Setup #5		
	C	F	D	C	F	D
A1	SW	3rd	S	SW	3rd	S
A2	SW	3rd	W	SW	3rd	W
A3	SW	4th	S	SW	4th	S
A4	SW	4th	W	SW	4th	W
A5	SW	2nd	S	SW	2nd	S
A6	SW	2nd	W	SW	2nd	W
A7	SW	1st	S	SE	1st	S
A8	SW	1st	W	SE	1st	W
A9	NW	1st	S	NE	1st	S
A10	NW	1st	W	NE	1st	W

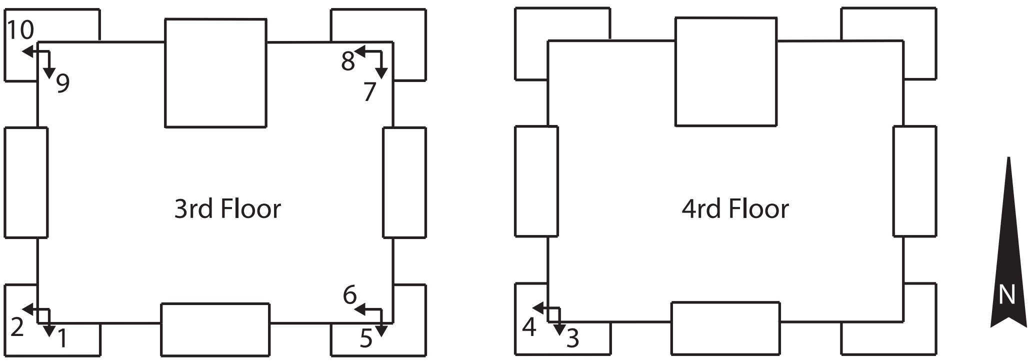

Figure 4: Accelerometer locations and directions for test setup #2

that the data are reliable. The damping estimates rise for these lower multipliers, and this may be genuinely showing that the damping ratio in this building rises for very low amplitudes. The purpose of this work is to provide data relevant to higher-amplitude wind-induced vibration, which might result in discomfort to building occupants, as well as pointers towards elastic properties for seismic design. It was therefore decided to use values at the upper end of the range of reliable data, which also corresponds to a region of consistent damping estimates. Thus, 1.5 times the standard deviation was chosen, which also corresponds to that recommended by Rodrigues and Brincker (2005) to give a balance between obtaining sufficient segments for averaging, and using data with amplitude above the noise floor.

5. Modeling

The behaviour of timber structural systems and their connections is substantially nonlinear. This is true at low loads as well as high, since mechanical timber connections using screws or nails require a certain level of force to overcome the initial slip in a connection, and mobilise its full stiffness. Detailed modeling of this behaviour is an important topic for future research into the serviceability behaviour of timber structures, but was beyond the scope of this study. Instead, a model of the more predictable concrete core was used to investigate the relative contribution of the concrete core and the timber structure.

A finite element model including both the concrete core and timber walls has been used for comparison with the measured mode shapes. There is not currently a suitable empirical basis to model the timber walls under serviceability loads, so the natural frequencies calculated by this model are not reported, but the mode shapes are presented to validate those measured experimentally.

The material properties used in modeling are given in Table 4. The layout of the structural walls in the core is shown in Figure 6, along with an image of the walls, modeled as shell elements in the finite element model. The second moment of area of the core is 1.97 m^4 about the north-south axis, and 6.30 m^4 about the east-west axis.

Using the building mass distribution given in Table 1, the natural frequency of the building in the two lateral directions was estimated using just the stiffness of the concrete core. The resulting natural frequencies are 2.4 Hz in the east-west direction, and 4.3 Hz in the north-south direction. These calculated frequencies form the basis of a the discussion, in Section 7, of the additional dynamic stiffness added by the timber shear walls.

Table 4: Material properties used for modeling (BSI, 2009, 2014)

Material	Elastic Modulus (N/mm^2)	Shear Modulus (N/mm^2)
Concrete	30000	12500
Timber	9000	560

A finite element analysis of the buildings was carried out using SAP2000 software. The concrete core and all timber walls were modeled by shell elements, and the elastic properties of the materials were as in Table 4. Both the shear deformation and the bending deformation of the walls was taken into account. Each floor was modeled with a diaphragm constraint. The masses given in Table 1 were applied, with the rotational mass at each level modeled.

6. Results

An example of the steps in the process used to extract the modal properties is shown in Figures 7 and 8. A 10-second segment of the 30-minute time-history of acceleration is shown in Figure 7, in which can be seen the periodicity in the response. The periodicity is brought out much more clearly by the averaging process of the random decrement method, as can be seen in the random decrement signature in Figure 8.

In the frequency domain, the peaks corresponding to two modes of vibration are evident. The ITD method uses this random decrement signature along with the other 9, which have been generated from the other accelerometers, using the same triggering times. Superimposed on the frequency spectrum in Figure 8, it can be seen that the ITD method identifies the two modes corresponding to clear peaks on this spectrum, and another, which may be more clearly seen on the spectrum from another accelerometer.

Three modes of vibration were identified for each building: two with predominantly lateral movement and one of torsional movement in plan. The mode shapes are shown in Figure 9, along with those calculated by finite element analysis. In one lateral mode, the concrete core is in the centre of the building in the direction of movement, while in the other, the core is off-centre. It is evident that the concrete core represents a particularly stiff part of the structure, since the mode shape with the core off-centre includes a substantial rotation around the core, with the greatest movement at the edge

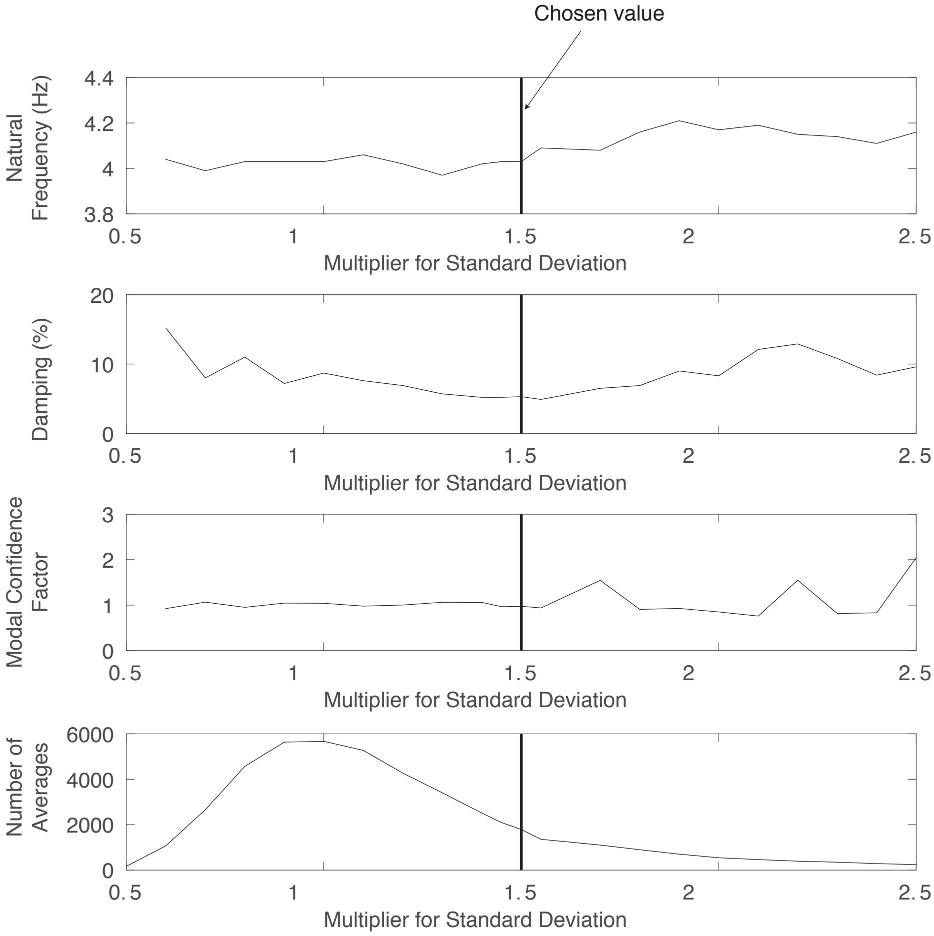

Figure 5: Study of the sensitivity of modal parameters to the chosen random decrement trigger level

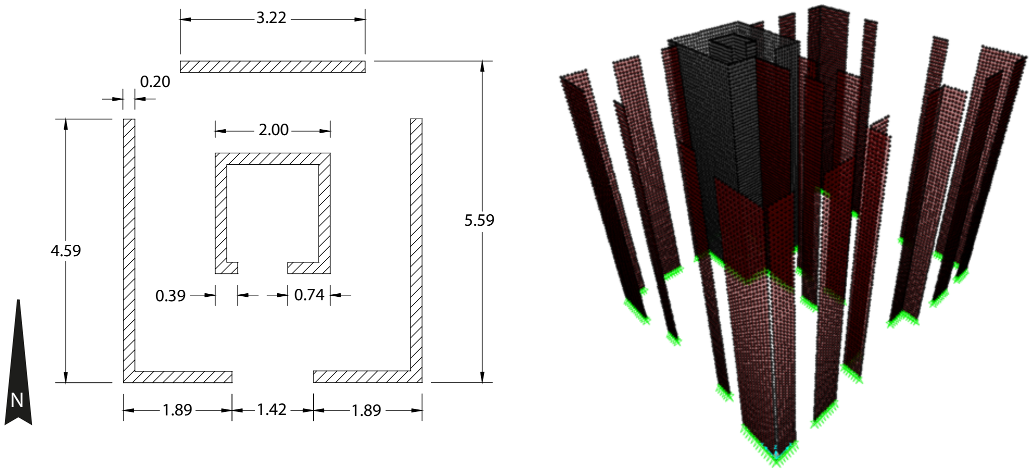

Figure 6: Layout of structural walls in the concrete core, with all dimensions in metres, and of the walls in the finite element model

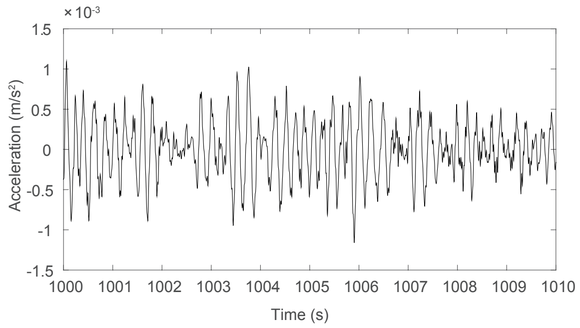

Figure 7: A 10-second segment of the time-history of acceleration recorded by accelerometer A1 for the CLT building

of the building opposite the core, and the least at the core itself.

The damping ratios and natural frequencies may be calculated separately from the results of each test, and so there is a range of natural frequency and damping estimates for each mode. The first mode was identified using accelerometer A2 as the trigger channel, and the following two modes using accelerometer A1. This was the same for the analysis of each building, with the exception of the second mode in the CLT building, which was difficult to identify due to its proximity to the third mode. It was found that using accelerometer A5 in test 1 as the trigger channel brought out the properties of this mode. Since this accelerometer location was not present in the other tests, it was only possible to draw a partial mode shape for this mode, as can be seen in Figure 9.

The reliability of the results for each mode from each test was assessed by calculating a modal confidence factor (MCF) (He and Fu, 2001), by using the same data with a time delay of 0.1 s to recalculate the residual for that mode, allowing for the exponential decay in its magnitude. For a true mode, the magnitude of the ratio of the two residuals should be close to unity. This ratio is the MCF. For the TF building, in the case of the fundamental mode, it was possible to use the first four tests, whereas for modes two and three, only the first two tests were suitably reliable.

The order of the modes is the same in each building: the two lowest-frequency modes consist predominantly of lateral movement, and the next one predominantly rotation. Table 5 shows the estimated modal properties using the ITD method for each building. The frequencies and damping ratios for the first two modes of vibration were very similar, while the TF building exhibited a higher frequency in the third, torsional, mode.

The significance of these results in terms of wind-induced vibration can be put into context based on a calculation using Eurocode 1 (BSI, 2005). According to this standard, the peak acceleration corresponding to a 1-year return period storm may be calculated, and this may be compared with criteria for human comfort given in (ISO, 2007). Based on the measured natural frequency and damping ratios, these buildings comfortably meet these criteria, with a peak acceleration of 0.024 m/s^2, well below the limit of 0.082 m/s^2 at this frequency. In Figure 10, the calculation has been extrapolated, with the natural frequency of the building assumed to be inversely proportional to the square of the height, as for a vertical cantilever with constant bending stiffness. The similarity in the properties of the fundamental mode in each building means that this calculation applies equally well to each timber structural system, and can be considered a statement about timber structural systems with a concrete core in general. Figure 10 shows that, using these structural properties, the limit for human comfort is quickly exceeded for even a few metres extra height over the 15.6 m of these buildings.

7. Discussion

The principal effect of the concrete core on the global response of the buildings is that it forms the stiffest part of the structure, and since it is asymmetrically located, it creates torsion in one of the modes of vibration. Figure 9 shows that, in Mode 1 for each building, the off-centre concrete core causes a torsional element to this predominantly lateral mode shape. In Mode 2, for which the building is symmetrical about the core, the mode shape is one of pure lateral sway.

Comparing the measured natural frequencies with those calculated using the core alone, it appears that the contribution of the timber shear walls is different in each of the lateral modes of vibration. In the first mode, a combination of lateral movement in the east-west direction and torsion, the stiffness of the core alone would suggest a natural frequency of 2.4 Hz, according to the calculation in Section 5. The real measured frequency of around 4.1 Hz therefore appears to include a substantial contribution from the timber walls, which is consistent with the substantial movement of the timber walls seen in the mode shape. In the second mode, for lateral movement in the north-south direction, the core alone was expected to give a natural frequency of 4.3 Hz. The measured natural frequency of around 4.9Hz is therefore only slightly higher. In this mode, the timber walls may only deform as far as the core, and so it appears they do not develop sufficient stiffness at that

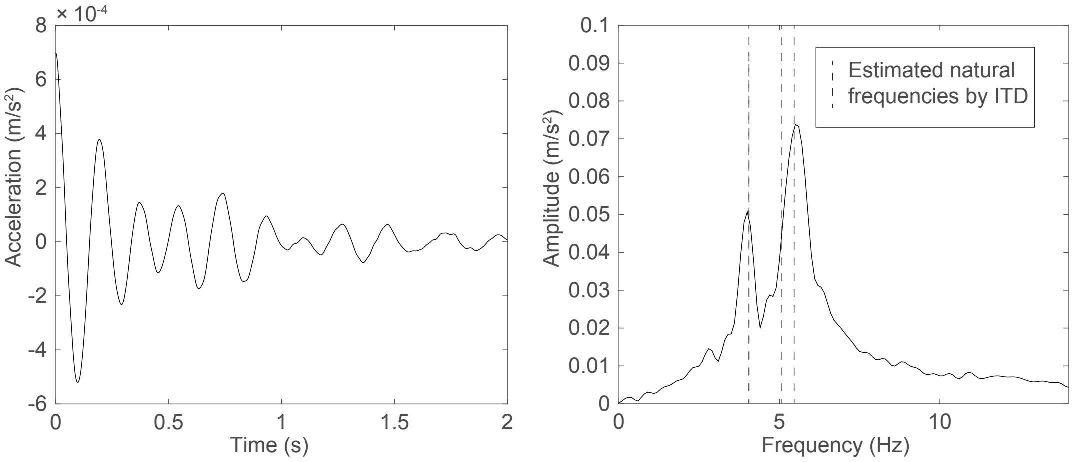

Figure 8: The random decrement signature for accelerometer A1 for the CLT building (left) and the Fourier transform of the random decrement signature, showing the modes identified by the ITD method (right)

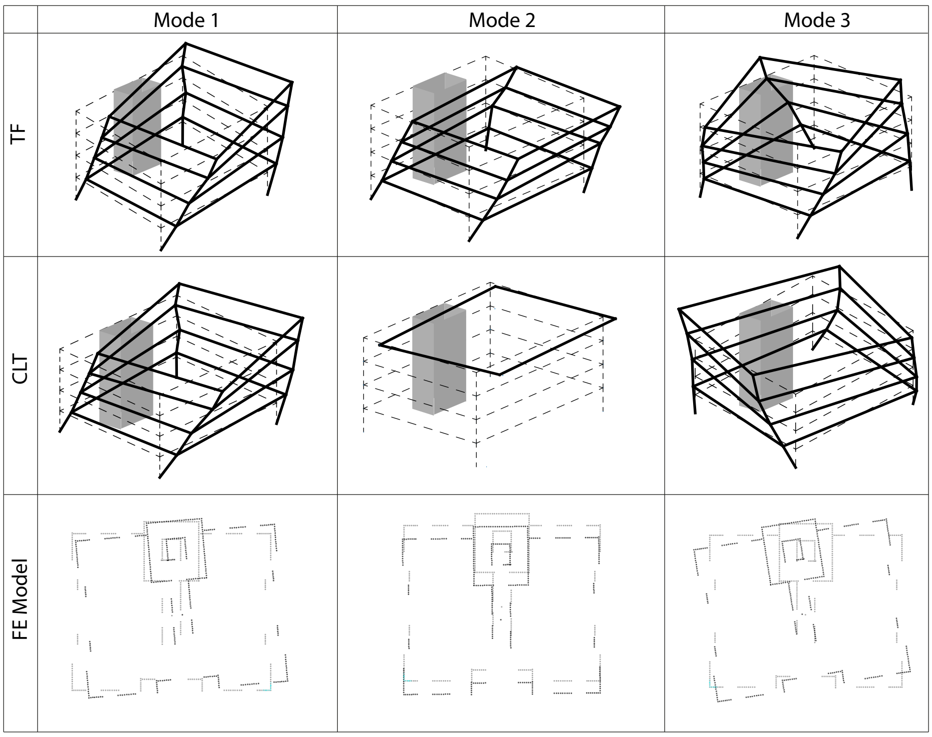

Figure 9: Mode shapes for each building, with the location of the concrete core indicated by the grey box for orientation, and mode shapes calculated from the finite element (FE) model, in plan

Table 5: Ranges of natural frequency and damping ratio for each building, and the range of root mean square (RMS) magnitude of acceleration, measured at the south east corner of the fourth floor, at which the parameters were measured

Mode	Natural Frequency (Hz)	Damping Ratio (%)	RMS (mm/s^2)
TF Building			
1	4.10-4.16	5.8-6.6	5.1-8.8
2	4.87-4.97	7.3-8.7	5.1-8.8
3	6.29-6.34	4.6-4.7	5.1-8.8
CLT Building			
1	4.03-4.08	4.9-6.9	2.5-10.2
2	4.80-4.99	6.5-8.3	2.5-10.2
3	5.51-5.63	4.7-7.0	2.5-10.2

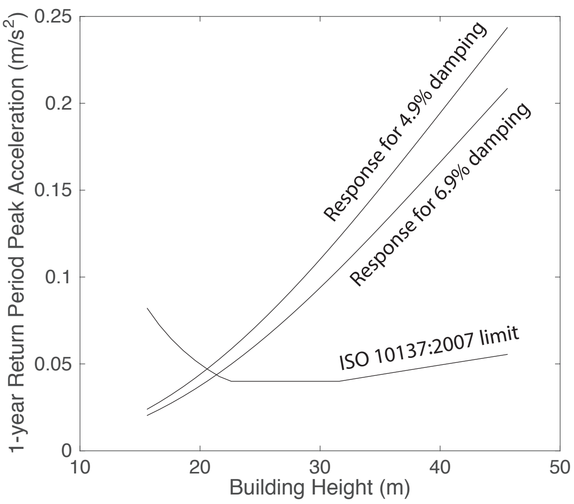

Figure 10: Extrapolating the calculation of peak acceleration for a taller building with the same structural system - this applies to either the CLT or TF building

amplitude to make a substantial contribution to the natural frequency. With the timber walls included with a representative elastic modulus, the mode shapes given by the finite element model show good agreement with those measured experimentally, as shown in Figure 9.

The similarity in the measured properties between the two buildings is notable. The solid timber panels of the CLT building, formed by stiff glue lines, might be expected to be more stiff in isolation than the sheathed stud walls of the TF building, with their more flexible nailed connections. In the complete structural system, however, there are various features described in Section 3 which may make the CLT building less stiff. The Timber frame building has a direct concrete-to-concrete connection from the wall to the floor, while the CLT relies on a screwed connection from the timber floor to a glulam beam loaded perpendicular to grain, and then bolted to the core - since the core is a particularly stiff element in the building, this may be expected to be a substantial difference It may also be that the insulation filling the voids in the TF panels causes them, under light wind loads, to behave as solid panels, and therefore provide a stiffness closer to that of the CLT panels.

The significant difference between the two buildings is in the frequency of the third mode of vibration. This mode is characterised by rotation of the building in plan, and it is possible that the greater mass of the walls in the CLT building, located further from the centre of rotation, is responsible for the lower frequency of this mode in that building.

The theoretical calculation of the response of these buildings to higher wind loads, and the response of taller buildings with the same system, shows the potential for wind-induced vibration to be a key design criterion in buildings of this form. These measurements represent to the authors' knowledge, the only damping measurements for timber buildings with a concrete core of five storeys or higher.

For seismic design, the key result is the mode shape in the fundamental mode, and the substantial torsional contribution it shows. This would produce high shear stresses in the core as it resists these torsional movements, and means that, while this design may be appropriate for these buildings, a more efficient design in a larger or taller building would be to place the concrete core or cores symmetrically in plan.

8. Conclusions

The dynamic properties of two buildings were identified using output-only modal analysis. The two buildings were identical in their layout and non-structural finishings, and differed only in the structural system of their timber shear walls.

The similarity in the dynamic response of the two buildings is striking, since their two structural systems transfer loads in such a different way: the TF building using a sheathed panel with a large proportion of void filled with non-structural insulation, relying on many mechanical connections between sheathing and studwork, in contrast to the the CLT building, in which the solid timber panels are formed from glued connections, with mechanical connections only at floor levels.

What the buildings have in common, however, appears more important than these differences in terms of their dynamic response: they are formed from the same structural material, and so the stiffness-to-weight ratio of the load-bearing material is similar, and loads are transferred between structural units by connectors, in the form of nails and screws, which resist load by embedding into the timber surrounding them.

Comparison of the measured natural frequencies with those which would be expected based on the core alone show that the contribution of the timber shear walls is greater in the first mode of vibration, where a torsional component means that the timber walls move at a greater amplitude than the concrete, allowing the full stiffness of connections to be mobilised. In the second mode, in which all of the structure moves laterally at the same amplitude, it appears that the low initial stiffness of the timber walls is evident, with little contribution to the dynamic stiffness of the building. This highlights a drawback in hybrid construction using systems with such different stiffness - the lower-stiffness part of the structure may not contribute fully to the overall stiffness of the structure.

There is a wide variety of timber structural forms, and sheathed stud walls and cross-laminated timber are currently important in the construction of multi-storey buildings. Further experimental research of this kind is required to develop the basis for estimation of damping and natural frequency in design.

Another key area of further work is to develop the modeling techniques necessary to predict the natural frequencies of this form of building. Such research may bring out the contribution of connections between timber and concrete elements in more detail, and would allow the extrapolation of findings to taller buildings by numerical modeling.

Acknowledgements

This collaborative research was made possible by funding from the European Union COST initiative, COST action FP1004, by funding from the BRE Trust, and by a Leverhulme Trust Programme Grant. The authors also acknowledge the assistance of the Trentino Social Housing Company ITEA.

References

Brownjohn, J., Magalhães, F., Caetano, E., Cunha, Á., 2010. Ambient vibration re-testing and operational modal analysis of the Humber Bridge. Engineering structures (Online) 32 (8), 2003–2018.

BSI, 2005. BS EN 1991-1-4:2005 Eurocode 1. Actions on structures. General actions. Wind actions. British Standards Institution.

BSI, 2009. Structural timber. Strength classes.

BSI, 2014. BS EN 1992-1-1:2004. Eurocode 2. Design of concrete structures.

Casagrande, D., Rossi, S., Sartori, T., Tomasi, R., 2015. Proposal of an analytical procedure and a simplified numerical model for elastic response of single-storey timber shear-walls. Construction and Building Materials.

Ceccotti, A., Sandhaas, C., Okabe, M., Yasumura, M., Minowa, C., Kawai, N., May 2013. SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated timber building. Earthquake Engineering & Structural Dynamics 42 (13), 2003–2021.

Cunha, Á., Caetano, E., 2006. Experimental modal analysis of civil engineering structures. Sound and Vibration 40 (6), 12–20.

Ellis, B. R., Bougard, A. J., 2001. Dynamic testing and stiffness evaluation of a six-storey timber framed building during construction. Engineering structures (Online) 23 (10), 1232–1242.

Filiatrault, A., Christovasilis, I. P., Wanitkorkul, A., van de Lindt, J. W., Sep. 2009. Experimental Seismic Response of a Full-Scale Light-Frame Wood Building. Journal of structural engineering 136 (3), 246–254.

Folz, B., Filiatrault, A., Apr. 2001. Cyclic Analysis of Wood Shear Walls. Journal of structural engineering 127 (4), 433–441.

Foti, D., Chorro, S. I., Sabbà, M. F., 2012. Dynamic Investigation of an Ancient Masonry Bell Tower with Operational Modal Analysis. The Open Construction and Building Technology Journal 6, 384–391.

Fragiacomo, M., Dujic, B., Sustersic, I., 2011. Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions. Engineering structures (Online) 33 (11), 3043–3053.

Gavric, I., Fragiacomo, M., Ceccotti, A., 2015. Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures. Materials and Structures 48 (6), 1841–1857.

Gsell, D., Feltrin, G., Schubert, S., Steiger, R., Motavalli, M., Jan. 2007. Cross-Laminated Timber Plates: Evaluation and Verification of Homogenized Elastic Properties. Journal of structural engineering 133 (1), 132–138.

H Cole, J. R., 1968. On-the-line analysis of random vibrations. In: 9th Structural Dynamics and Materials Conference. American Institute of Aeronautics and Astronautics.

He, J., Fu, Z.-F., 2001. Modal Analysis. Butterworth-Heinemann, Oxford.

He, M., Lam, F., Foschi, R. O., Aug. 2001. Modeling Three-Dimensional Timber Light-Frame Buildings. Journal of structural engineering 127 (8), 901–913.

ISO, 2007. Bases for design of structures - Serviceability of buildings and walkways against vibrations. International Organization for Standardization.

Judd, J. P., Fonseca, F. S., Feb. 2005. Analytical Model for Sheathing-to-Framing Connections in Wood Shear Walls and Diaphragms. Journal of structural engineering 131 (2), 345–352.

Magalhães, F., Caetano, E., Cunha, Á., Flamand, O., Grillaud, G., 2012. Ambient and free vibration tests of the Millau Viaduct: Evaluation of alternative processing strategies. Engineering structures (Online) 45 (0), 372–384.

Reynolds, T., Harris, R., Chang, W.-S., Bregulla, J., Bawcombe, J., 2015. Ambient vibration tests of a cross-laminated timber building. Proceedings of the ICE - Construction Materials.

Rodrigues, J., Brincker, R., 2005. Application of the random decrement technique in operational modal analysis. In: 1st Int. Operational Modal Analysis Conf. (IOMAC). Aalborg Universitet, Aalborg, Denmark.

Tomasi, R., Sartori, T., Casagrande, D., Piazza, M., Dec. 2014. Shaking table testing of a full scale prefabricated three-story timber framed building. Journal of Earthquake Engineering, 141217131810008.

van de Lindt, J. W., Pryor, S. E., Pei, S., 2011. Shake table testing of a full-scale seven-story steel-wood apartment building. Engineering structures (Online) 33 (3), 757–766.

Vessby, J., Enquist, B., Petersson, H., Alsmarker, T., 2009. Experimental study of cross-laminated timber wall panels. Holz als Roh- und Werkstoff 67 (2), 211–218.